APPENDIX D

Evaluating Older Powerhouses

D-1. General.

a. Many Corps powerhouses date back to the early 1900s. Therefore, it is important that engineers evaluating the seismic vulnerability of these older powerhouses understand aspects related to:

- Concrete and reinforcing steel material properties used in their design, and
- Codes and guidance governing their design.

b. The year in which a Corps powerhouse was constructed will have significant influence on how it will perform when subjected to earthquake ground motions. Historical information on material properties valuable in assessing seismic performance is provided in FEMA 356 (2000). Default strength and yield properties from FEMA 356 (2000) are useful in the preliminary seismic assessment of older Corps powerhouses. Additional information may be available on the contract drawings or in the contract specifications. In some instances, sampling and testing will be required to confirm that strengths are the same as those originally assumed for preliminary seismic evaluations.

c. Older powerhouse walls will likely not have development and splice lengths that comply with current code (ACI 318-02) requirements. In addition, powerhouses constructed before 1947 are unlikely to have the “high-bond” deformation patterns typical of modern reinforced concrete structures. Information on the yield and tensile strength properties of older reinforcing steel is provided in FEMA 356 (2000). This appendix provides guidance on one approach that can be used to assess the strength of older powerhouse walls that do not have adequate splice and development lengths. This deficiency can be the result of either past code design practice or the “low-bond” deformation pattern of older reinforcement. The information contained here on low-bond reinforcement is based on FEMA 356 (2000) and CRSI (2001).

d. In most instances with older powerhouses, it will be difficult to determine the concrete compressive strengths and reinforcing steel strengths. The default lower-bound values provided in FEMA 356 (2000) and repeated in this appendix are intended for use in performing the LSP, LDP, and Special Analyses of existing powerhouse superstructures. In cases where demand-to-capacity ratios (DCR) are marginal with respect to meeting acceptance criteria, it may be advisable to conduct destructive and non-destructive testing to determine in-place concrete strengths and the yield strength and ultimate tensile strain capacity of the reinforcing steel. With older low-bond reinforcement, pull-out testing may be useful for determining the adequacy of the splice and development lengths used in the construction of critical components.

e. Component strength will be a function of displacement ductility demand, with concrete shear strength declining rapidly as displacement ductility demand increases. Older powerhouse superstructure components do not contain the confinement steel required by modern codes. Without adequate confinement, the ability of the tension reinforcement to develop its ultimate capacity also declines with increased displacement ductility demand.

D-2. FEMA 356 (2000) Ductility Demand Classifications.

a. FEMA 356 (2000) defines three classifications of displacement ductility demand. They are:

- Low ductility demand
- Moderate ductility demand
- High ductility demand.

b. The ranges of ductility demand for each classification, per FEMA 356 (2000), is as indicated in Table D-1.

Table D-1. Component ductility demand classifications.

Maximum value of displacement ductility demand or flexural response DCR	**Classification description**
< 2	Low ductility demand
2 to 4	Moderate ductility demand
> 4	High ductility demand

c. FEMA 356 (2000) requires that deformed straight bars, hooked bars, and lap-spliced bars in yielding regions of components with moderate or high displacement ductility demand meet the splice and development requirements of Chapter 21 – Special Provisions for Seismic Design, ACI 318-02. Deformed straight bars, hooked bars, and lap-spliced bars in yielding regions of components with low displacement ductility demand can meet the splice and development requirements of Chapter 12 – Development and Splices of Reinforcement, ACI 318, except that requirements for lap splices shall be the same as those for straight development of bars in tension without consideration of lap splice classifications. In most cases for powerhouse superstructure walls, the displacement ductility demands due to earthquake ground motions will be low. The tensile capacity of the reinforcement may need to be reduced for those older powerhouse walls that fail to meet the above code-specified splice and development length requirements. FEMA 356 (2000) guidance for this is provided in the paragraph below.

D-3. FEMA 356 (2000) Requirements for Nonconforming Splice and Development Lengths.

a. Splice length requirements.

(1) Longitudinal reinforcement splices are almost always located at the base of a wall or column where plastic hinging is likely to occur when seismic moment demand exceeds the nominal moment capacity of the wall or column. Walls and columns of existing powerhouses are almost always non-conforming (NC) per FEMA 356 (2000) because:

- Transverse confining reinforcement, if present, usually has spacings that are greater than one-third the depth of the member.

- The strength provided by the transverse reinforcement is less than three-fourths the shear capacity of the member.

(2) Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements in the code provisions specified above, the capacity of the existing reinforcement shall be calculated using the following equation:

$$f_S = \frac{l_b}{l_d} f_y \tag{D-1}$$

where:

f_S = maximum stress that can be developed in the bar for the straight development, hook, or lap splice length (l_b)
l_b = splice length provided
l_d = length required by ACI 318 Chapter 12, or 21 as appropriate for straight development, hook development, or lap splice length, except required splice lengths may be taken as straight bar development lengths in tension.

(3) Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth of the component, it shall be permitted to assume that the reinforcement retains the calculated maximum stress to high ductility demands.

(4) FEMA 356 (2000) also indicates: "For larger spacings of transverse reinforcement, the development stress shall be assumed to degrade from f_S to 0.2 f_S at a ductility demand equal to two."

(5) This degradation need not be considered for powerhouse superstructure components if they have axial load ratios (*ALR*) less than 0.15, which is commonly the case:

$$ALR = \frac{P}{A_G\, f'_{ca}} \leq 0.15 \tag{D-2}$$

where:

P = axial load on wall or column
A_G = gross sectional area of wall or column
f'_{ca} = actual compressive strength of concrete.

(6) In research performed on concrete columns (Watson, Zahn, and Park 1994), it was determined that: "At low axial load ratios (<0.15) extremely large curvature-ductility factors are available with only very small quantities of confining reinforcement steel. In such cases, the

amount of transverse reinforcement required is not governed by the requirements of concrete confinement."

(7) In addition, it has been observed that when compressive strains are below 0.2 percent (0.002), the chance for micro cracking and bond deterioration that could lead to reinforcing steel splice failure is low [see Appendix G, Strom and Ebeling (2005)].

b. Embedment length requirements.

(1) The strength of deformed, straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than three bar diameters ($3d_b$) shall be calculated as follows:

$$f_s = \frac{2500}{d_b} l_e \le f_y \qquad \text{(D-3)}$$

where:

f_s = maximum stress (psi) that can be developed in an embedded bar having an embedment length l_e (in.).
d_b = diameter of embedded bar (in.).

(2) FEMA 356 (2000) also indicates: "When f_s is less than f_Y, and the calculated stress in the bar due to design loads equals or exceeds f_s, the maximum developed stress shall be assumed to degrade from f_s to 0.2 f_s at a ductility demand equal to two."

(3) For reasons stated above, this degradation need not be considered for powerhouse superstructure components that have axial load ratios less than 0.15.

D-4. Splice and Development Length Requirements for "Low-Bond" Deformation Bars.

a. In the early 1900s, the reinforcing steel could consist of:

- Plain round bars
- Twisted square bars
- Round and square bars with "low bond" deformations.

b. Many of these early bars were patented or part of patented reinforcing systems. The term "low bond" is used to distinguish these bars from the "high bond" deformation type of reinforcing steel that became commonplace in 1947 and is basically unchanged to the present day (CRSI, 2001). Information useful to the evaluation of older reinforced concrete structures can be found in CRSI (2001).

c. CRSI (2001) states that: “For older structures, it is prudent to consider all varieties of reinforcing bars—plain round, old style deformed, twisted square, and so on—conservatively and simply as 50 percent effective in bond and anchorage as current bars. In other words, the tension development lengths, l_d , for the old bars would be twice (double) the l_d required for modern reinforcing bars. Since most strength design reviews for flexure will be based on a yield strength, f_Y = 33,000 psi instead of today’s 60,000 psi, the tension development lengths for the old bars can be determined by adding 10 percent to any current table of tension development lengths, l_d , for modern reinforcing bars.”

d. FEMA 356 (2000) is more tolerant with respect to older square reinforcement that is twisted, allowing the development strength to be as specified for deformed bars in ACI 318-02. In the ACI 318-02 computations, an effective round bar diameter is determined based on the gross area of the square bar. Square straight bars, however, are to be treated as plain bars using the CRSI (2001) process described above. FEMA 356 (2000) permits higher development strengths for bars classified as “plain” if they can be justified by approved tests or calculations that consider only the chemical bond between the bar and the concrete.

e. Older square bar reinforcements with areas equivalent to the modern round No. 14 and 18 bars may exist in some older powerhouse superstructures. These bars will be lap spliced rather that welded or mechanically connected as required in modern reinforced concrete structures. It is suggested that these bars be treated as 50-percent effective in bond per the CRSI (2001) recommendations provided above.

D-5. <u>Default Values for Use in LSP, LDP, and Special Analyses</u>.

a. Table D-2 provides tensile and yield properties of reinforcing bars for various years. Table D-3 provides tensile and yield properties of reinforcing bars for various ASTM designations.

Table D-2. Default lower-bound tensile and yield properties of reinforcing bars for various periods.[1] [After Table 6-1, FEMA 356 (2000).]

Year	Grade	Structural[2] 33	Intermediate[2] 40	Hard[2] 50	60	70	75
	Min. Yield (psi)	33,000	40,000	50,000	60,000	70,000	75,000
	Mix. Yield (psi)	55,000	70,000	80,000	90,000	95,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1974-1987			x	x	x	x	
1987-Present			x	x	x	x	x

Notes:
1. An entry of “x” indicates the grade was available in those years.
2. The terms structural, intermediate, and hard became obsolete in 1968.

Table D-3. Default lower-bound tensile and yield properties of reinforcing bars for ASTM specifications and periods.[1] [After Table 6-2, FEMA 356 (2000).]

				Struct.[2]	Inter.[2]	Hard[2]			
			ASTM Grade	33	40	50	60	70	75
			Min. Yield (psi)	33,000	40,000	50,000			
ASTM Desig.[5]	Steel Type	Year Range	Min. Tensile (psi)	55,000	70,000	80,000	90,000	95,000	100,000
A15	Billet	1911-1966		x	x	x			
A16	Rail[3]	1913-1966				x			
A61	Rail[3]	1963-1966					x		
A160	Axle	1936-1964		x	x	x			
A160	Axle	1965-1966		x	x	x	x		
A408	Billet	1957-1966		x	x	x			
A431	Billet	1959-1966							x
A432	Billet	1959-1966					x		
A615	Billet	1968-1972			x		x		x
A615	Billet	1974-1986			x		x		
A615	Billet	1987-1997			x		x		x
A616[4]	Rail[3]	1968-1997				x	x		
A617	Axle	1968-1997			x		x		
A706	Low-Alloy	1974-1997						x	
A955	Stain-less	1996-1997			x		x		x

Notes:
1. An entry of “x” indicates the grade was available in those years.
2. The terms structural, intermediate and hard became obsolete in 1968.
3. Rail bars are marked with the letter “R.”
4. Bars marked “s” (ASTM 616) have supplementary requirements for bend tests.
5. ASTM steel is marked with the letter “W.”

b. Concrete properties and strength are also dependent on the time frame in which construction occurred. Many older structures are not air entrained and therefore may have suffered freeze-thaw deterioration. A condition assessment is always an important part of any seismic evaluation. Table D-4 provides lower-bound compressive strengths for structural concrete for various time periods.

Table D-4. Default lower-bound compressive strength of structural concrete (psi). [After Table 6-3, FEMA 356 (2000).]

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900-1919	1000-2500	2000-3000	1500-3000	1500-3000	1000-2500
1920-1949	1500-3000	2000-3000	2000-3000	2000-3000	2000-3000
1950-1969	2500-3000	3000-4000	3000-4000	3000-4000	2500-4000
1970-Present	3000-4000	3000-5000	3000-5000	3000-10000	3000-5000

c. Probable vs. lower-bound strength. The probable strength of materials used in construction is generally greater than the lower-bound strength values used for design. FEMA 356 (2000) provides information to relate expected strengths of concrete and reinforcing steel to their lower-bound design basis values. This information is provided in Table D-5.

Table D-5. Factors to translate lower-bound material properties to expected strength material properties. [After Table 6-4, FEMA 356 (2000).]

Material Property	**Factor**
Concrete compressive strength	1.50
Reinforcing steel tensile and yield strength	1.25
Connector steel yield strength	1.50